WENDLING

Review of Detroit Avenue concrete arch bridge at Cleveland, Ohio

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REVIEW OF DETROIT AVENUE CONCRETE ARCH BRIDGE AT CLEVELAND, OHIO

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This is to certify that the thesis prepared under my supervision by JACOB WENDLING entitled Review of the Detroit Avenue Concrete Arch Bridge at Cleveland, Ohio, is approved by me as meeting this part of the requirements for the degree of Bachelor of Science in Civil Engineering.

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REVIEW of DETROIT AVENUE CONCRETE ARCH BRIDGE

at

CLEVELAND, OHIO.

The Detroit Avenue Concrete Arch Bridge at Cleveland, Ohio, now under construction, over the Rocky River, is notable for its great length of main arch and for its methods of construction. It is the purpose of this thesis to give a brief description of this structure, which contains the largest concrete arch ever undertaken, either in this country or in Europe; and to describe the methods of construction as far as the work has progressed up to this writing. In doing the latter the writer has the advantage which comes from personal experience, having been in the employ of the contractors during the summer vacation of 1909. It was on account of this connection with the work that this subject was chosen for a thesis.

I. DESCRIPTION of BRIDGE

The bridge carries Detroit Avenue over Rocky River about half a mile above its outlet into Lake Erie. It connects together Cleveland's two western suburbs, Lakewood and Rocky River, which are rapidly becoming first class residence districts. The river runs in a deep ravine with steep wooded slopes. The location of the bridge is such as to call for a structure of an ornamental character. With this consideration concrete was de-

cided upon as the most suitable material for the purpose.

The bridge was designed and its construction is being supervised by Mr. A. M. Felgate under the direction of the engineer of Cuyahoga County. The river is subject to heavy floods and ice jams early in the spring, and therefore it was desirable to obstruct the stream as little as possible, and also to reduce the danger of damage to the bridge to a minimum. This led to the consideration of alternate plans with one and two channel spans. An estimate of the cost showed that the material saved with one large span nearly offset the increased cost of building it, besides affording an opportunity of erecting the longest concrete arch in the world.

The total length of the bridge is 708 feet, consisting of a main arch of 280 feet in the clear and five 44 foot approach arches. The west approach has three spans, while the approach on the steeper east slope has two. To preserve the architectural balance of the whole the east abutment is made longer than the west. The bridge has a width of 60 feet 4 inches over all and a clear width of 56 feet, giving a 40 foot roadway with two car tracks, and two 8 foot sidewalks. The roadway is 95 feet above ordinary water level, and has an ascending grade to the east of 1 1/2 per cent. The floor is carried over the haunches of the main arch by four spandrel arches on each side of the crown. As the springing points of the main arch are at the same elevation, the spandrel piers on one side are higher and, therefore heavier, putting extra weight on that side, which is counterbalanced by making the piers on the other side four inches thicker.

Fig. 1, Plate 1, shows an elevation of the bridge, see page 31.

The type of the structure selected is the same as that of the Walnut Lane Bridge at Philadelphia, which consists practically of two arches side by side with the space between floored over. This twin construction is economical of material and forms. The two halves can be built independently and the forms of the first used for the second. The economy of this type is apparent in the main span of the Rocky River Bridge which consists of two ribs each 22 feet wide, and 11 feet thick at the springing line and 18 feet wide and 6 feet thick at the crown, giving a supporting width 60 per cent of the floor width. The two ribs are 16 feet apart at the crown, and the floor overhangs 4 feet on each side.

II. CONSTRUCTION

Bids for the work were received in August, 1908. The bid of $208,300.00 by Schillinger Brothers of Columbus, Ohio, was the lowest; and they were awarded the contract. The erection of the plant and excavations for the foundation were started October 29, 1908. The excavation was through 2 or 3 feet of clay and into the shale about 19 to 21 feet. The excavated material was loaded into skips handled by a boom derrick, and deposited within reach of the derrick along the shore on the down stream side of the bridge. The main piers are close to the water's edge, but no coffer-dams or sheet piling were necessary, very little water entering the excavation. The water that did enter came through the clay. At first the water was allowed to go

to the bottom of the excavation and was from there pumped out with a No. 2 Emerson pump. After it was found that no water filtered in through the laminations in the shale, the clay excavation was widened all around so as to leave a berm of shale on which to collect the water and prevent it from running into the workings below. The footings for the main piers were carried down 23 feet, and exploration holes were drilled several feet further to make sure that the foundations were safe.

The work was continued throughout the winter of 1908-09. The first concrete was placed December 5th, 1908. Concreting was done by two gangs, one on each side of the river. The concrete plant was established on the east bank of the river. The mixer was of the continuous type with a capacity of about 200 yards in 10 hours. It was elevated enough to permit the discharge of concrete into buckets carried on a flat narrow-gage car. The stone and sand bins were constructed above the mixer, and were kept supplied from storage piles back of the mixer with a clamshell bucket operated from a 70-foot boom derrick. This derrick also took the stone and the sand from where it was dumped by the wagons and deposited it on the storage piles. The cement, from the cement shed alongside of the mixer, was hoisted to the mixing platform by the same derrick. The concrete was handled in buckets by a 900-foot Lidgerwood cableway carried by 65-foot movable towers. The east tower contains the engine and drums for operating the cableway. Figure 3, Plate III, Page 34, shows the cableway towers with the mixer just to the left of it. The stiff leg of the derrick is seen over the top of the mixer. In the foreground

the forms for the subway floor are partially in place. The concrete buckets varied in size from 3/4 to 1 1/2 yards. The narrow-gage track carrying the concrete buckets from the mixer to the cableway ran parallel to and in front of the track of the cableway towers so that the buckets could be brought under the cableway when in any position. The steam for the mixer and the cableway was furnished by three boilers located just back of the tower. The concrete materials had to be hauled by team about half a mile from the railway station to the mixer. The sand and the stone were unloaded from the cars by a clamshell bucket which dumped into an elevated bin from which it was drawn into the wagons. The only hand shoveling required was in cleaning out the cars. The cement all had to be hauled twice; first from cars to the storage sheds at the station where it had to remain 30 days awaiting the results of the county engineer's tests, and second to the mixer or to the shed at the mixer. In hauling cement from the storage sheds to the mixer each team hauled 10 loads of 60 sacks per day. The average day's run of concrete while filling the foundations was 125 cubic yards, and the maximum nearly 200 yards.

The main piers were built in 16-foot sections. In constructing the first pier on either side of the river a platform (see Fig. 5, Plate IV, page 35) was placed over the hollow centre of the forms to receive the concrete from the cableway. From these six men shoveled the concrete into vertical chutes attached to the sides and ends of the platform. These chutes extended down into the forms to within a short distance of the concrete in place,

and they were cut off as the concrete rose in the form. In the forms a number of men spread the concrete and kept it well spaded. All exposed surfaces of the bridge are given a one-inch facing of one part cement, two parts sand, and two parts crushed granite, which is put in behind facing irons as the work progressed. The facing concrete was mixed by hand in mortar boxes on the east approach. On the piers the facing boxes were placed over the spaces between the pilasters, and the material let down to the men in coal scuttles. This method required about twelve men on a pier. In this manner the south main pier on each side of the river was run up to the floor of the bridge, the carpenters building a section of forms on one pier while the concrete men were filling a section on the other. Fig. 4, page 35, shows one of the piers nearly completed. After these two piers were built derricks with 50-foot booms were placed on them. Then instead of dumping the concrete onto the platform, in building the other two piers, the buckets were set on the platform by the cableway and then taken up by the derrick and dumped directly into place. This saved all the shoveling on platform and a large part of the spreading in the forms. There were four concrete buckets used on the work, which varied in size from 3/4 to 1 1/2 yards. The 3/4-yard bucket could be let down into the forms and dumped without dropping the concrete any appreciable distance, but with the larger buckets it was more convenient to simply take them over the place where the concrete was to be put and drop it down from the top. No separation of materials could be noticed in dropping a cubic yard or more in this manner a distance of 16 feet. The

specifications permitted the use of one man stones in this part of the work, but only in a few instances were any put in. No reinforcement is used in the piers except near the top, where a few bars are laid horizontally to distribute the thrust of the approach and spandrel arches.

In building the forms for the piers as much of the work as possible was done in the framing yard. Fig. 6, Plate V, shows how complicated the outer forms are. The two sides containing the pilasters are vertical, while the pilasters themselves have a batter of 1 inch in 40 on all three sides, and the other two sides also have a batter of 1 in 40. The core is 11 by 15 feet and without batter. The various parts of the 16-foot sections were built in units which could be easily handled with the derrick in erecting as well as in taking down. Two sets of forms were built, one for each side of the river, but owing to the batter, making each section smaller than the one beneath it, most of the forms were used only twice, once on each pier. In taking the forms down the bolts were first taken out, then the 8 by 8 inch or 8 by 10 inch timbers were either taken to the ground or directly onto the other pier. The three-sided piece forming the recess between the the two pilasters on either side of the pier was designed to be taken down as a unit, but it was found impossible to get it out in one piece, and all the 3 by 6 inch pieces in the middle part had to be sawed in two and the section taken out in two parts. All core forms had to be taken down in single pieces. As a rule a section of forms was not taken down until it was needed on the adjacent piece. The outer timbers were spaced

about 4 feet vertically. When necessary wedges were used between the timbers and the 3 by 3 inch uprights. Very little trouble was experienced due to forms bulging or jarring out of place in filling. All exposed surfaces faced with crusted granite are to be bush hammered or washed with acid to remove the division lines between succeeding layers of concrete and to remove the cement from the surface so as to give the whole structure a uniform granite appearance. The framing yard was just south of the concrete plant. Here all lumber was received and stored. On a 40 by 60 foot platform the form panels were laid out and built. A shed about 35 feet wide and 50 feet long contained a cross-cut saw, a rip saw and a band saw, each driven by a separate motor by power furnished by a local power company.

The number of men employed by the contractor during the last summer varied from about 65 to 80, distributed about as follows.

1 superintendent.

1 engineer.

1 timekeeper.

1 general foreman.

2 carpenter foremen.

2 concrete foremen.

1 foreman unloading material.

7 engineers and firemen.

15 carpenters.

1 blacksmith.

6 teamsters.

4 handy men.

24 concrete men.

7 men at mixer.

4 men unloading material.

The inspection and surveying force consisted of two inspectors, a resident engineer, an instrument man and two chainmen.

MAIN ARCH CENTRES and METHODS of CONSTRUCTION.

The centering for the main arch consists of two three-hinged trusses spaced 23 feet center to center. In the Walnut Lane Bridge wooden centers were used, and the only other arch in which a steel center has been used is the Delaware River Bridge on the D. L. & W. R. R. at Portland, New Jersey. The centering rests on concrete brackets built on the abutments of the main pier below the water level. These brackets are reinforced with old steel rails, and form a shelf extending the full width of the bridge. Fig. 9, Plate VIII, page 39, shows the centering during construction with the transverse and longitudinal rows of I beams not yet in place. Fig. 7, Plate VI, page 37, shows the top chords and those I beams; and Fig. 12, Plate X, page 41, shows a panel point looking up from underneath. This also shows the I beams and tension braces. These 20-inch I beams are to be used in reinforcing the roadway of the bridge, and are used in the centering to save as much other steel as possible, because it is not probable that the contractors will realize anything on it at the completion of the work. All field connections are made with bolts to facilitate

removal. The lower chords were cambered two inches and it was expected that enough deflection would occur to straighten them under load. The centering is supported on the concrete shelf by means of four sets of iron wedges, one set under each support. After the completion of the first rib these wedges were slacked off, permitting the whole center to lower about 3 inches. Then by means of block and tackle it was moved over bodily into place for the second rib. Fig. 13, Plate X, page 41, shows the first rib completed and the center in place ready for the second. Steel centering was used partly because of the length of the span, and partly because it was thought only one rib would be built in a season, which would expose a wooden centering to the ice and floods of the spring and winter of 1909-10. The work progressed faster than had been anticipated and both ribs were finished in the fall of 1909.

The forms on the steel center are shown in Fig 7, Plate VI, page 37, Timber bows on the I beams vary from 12 by 12 inches at the skewbacks to 3 by 12 at the crown, and are spaced 2 feet center to center. These bows were butt jointed to carry part of the load down to the footings. Under keys B, C and D, Plate VII, the bows were provided with relieving wedges to provide for deflections in the steel center. On top of these bows the intrados curve was formed by 2-inch furring strips cut to the radius of the arch and spiked to the bows. The lagging was 2 by 4 inch dressed on two sides and beveled to fit the curve of the arch. The side forms were constructed of 2-inch dressed and matched lumber with 3 by 6 inch and 4 by 8 inch studding set 30 inches

center to center, supported by 10 by 10 inch timbers running parallel to the arch ring. These timbers were held in place by 6 by 8 inch posts set about 10 feet center to center and held in place at the bottom by 5/8 inch iron stirrups hooked over the fangs of the top chord, and fastened across the top of the arch ring by 3 by 10 inch timbers bolted to pairs of opposite posts.

In assembling the side panels of the forms for the rib, great care was taken in turning the intrados and extrados curves. A platform 40 by 60 feet was provided in the framing yard on which the curves were laid out with a transit using 5-foot chords. Nails were driven at these 5-foot stations and a thin batten sprung around them. The curve was then marked on the platform along this batten.

To provide for slight deflections in the steel center during the application of the load, the concrete was placed in sections as shown in Fig. 8, Plate VII. Corresponding sections in each half of the rib were placed simultaneously. The top section on either side of the crown was placed first, and the others as numbered in Fig. 8, Plate VII. 4-foot keys were left between adjacent sections. The sections were held in place by three reinforced concrete strutts through each of the four lower keys.

The concrete in the ribs is of a 1 : 2 : 4 mixture with large sand stone slabs imbedded in it, thus forming a sort of rubble masonry. These stone slabs varied from 6 to 10 inches in thickness and in surface area from 10 to 35 square feet, and were set radially in the arch to take care of the vertical shear. For the lower sections the concrete was mixed as dry as possible

without requiring tamping so as to make it stand up on radial lines to receive the rubble stones. As the work progressed the angle of the radius increased and the concrete began to run to the bottom of the forms making this method of working slow and difficult. It was then decided to place the slabs in first, spacing them with wooden blocks and running the concrete in wet afterward. This method worked very well and was considerally faster than the other. A few inches of concrete were first placed in the bottom of the forms on which the stones were then set and properly spaced by blocking them up at the top. A wet mixture of concrete was then dumped on them and well spaded to fill all openings. As the concrete filled the joints the pressure on the wooden blocks was relieved and they were easily knocked out by a few taps of a shovel. The stones were set up in transverse rows about eight inches apart, and no stones in the same row were spaced closer than four inches. Joints were broken as much as possible. All stones were thoroughly cleaned and wetted before being placed in the arch. Each day's work was cut off in a plane normal to the compressive stresses in the arch by building a bulkhead across the forms.

The keys were filled in shortly after the voussoirs had been completed. The lower keys were filled first and the one at the crown last, a wet mixture being used in all except the crown key. As it had been decided to strike the centers as soon as possible and get the second rib under way, a 1 : 1 : 2 mixture, which would develop sufficient compressive strength in a short time, was used. The concrete in the top key was mixed quite dry

and thoroughly cammed into place. The first concrete was placed in the arch August 6th, 1909 and the top key finished September 9th, 1909. The concrete in this key was nineteen days old when the centers were struck.

On lowering the wedges the arch settled under its own weight 0.46 inch at the crown and 0.07 inch at points 10 feet on either side of the crown. A profile of the lagging was taken before lowering the centering to ascertain how near the intrados corresponded to the true curve. The crown was exactly right and the average variation was less than half an inch. Instrument readings on the steel centering showed that it deflected about 1 3/4 inches at the crown and bulged outward slightly at the haunches under the load of section 1. This condition was corrected as the succeeding sections were placed, the haunches lowering as the trusses lost their camber and the center pin rising until at the completion of section 6 the center pin had risen an inch and the lower chords had lost their camber entirely. When section 4 was being placed it was noticed that the lagging beneath section 2 and the lower half of section 3 was entirely free from the concrete, due to the camber coming out of the trusses under load, but section 2 at no time showed any tendency to tip forward on the skewback. Readings were taken frequently on the center pin and panel points of the centering to observe its action under varying conditions of loading and temperature, and it was decided to leave a joint on each side of the crown key to permit a slight rise of the center pin with higher temperature without breaking the concrete. The concrete surfaces of the voussoirs on each side of the crown key were therefore

given a coat of oil to prevent any bonding with the other key. The temperature remained fairly constant during the first two days after filling this key. Then with a rise in the temperature the centering rose and opened the joint on the west side of the key about 1/4 inch. This joint remained open more or less until the centers were struck. Each rib contains 2150 cubic yards of rubble concrete weighing approximately 4000 tons.

APPROACH ARCHES and FLOOR of BRIDGE.

The approach arches are 50 feet center to center of piers and 44 feet in the clear, the piers being 6 feet thick under the coping. The arches are semicircular and consist of four ribs each. The dimensions of the ribs are, 4 feet wide by 2 1/2 feet deep at the springing line and 3 1/2 feet wide by 1 1/2 feet deep at the crown. A small amount of reinforcement is used in the intrados at the crown and in the extrados over the haunches. Fig. 11, Plate IX, shows the detail drawings of the centers for these arches, and Fig. 10, Plate VIII, shows the centering and forms in place. The section through the forms shows the bottom lagging cut to the exact width of the ribs and the side forms set against it without cutting the intradosal curve in the side panels. In erecting the forms it was m ch more convenient to have the lagging projecting on each side and the intradosal curve cut into the side panel so that they could be set directly onto the lagging. This method was adopted and can be seen in Fig. 10, Plate VIII.

Fig. 2, Plate II, shows a cross section of the floor through the crown of the main arch and through the crown of an approach arch. The spaces over the crown of the main arch and between the ribs of the approach arches are subways extending the full length of the bridge. These subways are to take all underground pipes and wires which may be carried across the bridge. Section AA shows the spacing of the I beams which span the middle portion and carry the car tracks. The rails are fastened directly to these I beams. The floor of the bridge has a 1/2 inch expansion joint over each pier.

III. CALCULATION of STRESSES in MAIN ARCH RIBS.

The concrete in the main arch ribs was assumed to weigh 160 lbs. per cubic foot, and all other concrete 150 lbs. per cubic foot. The safe compressive strength of the concrete was assumed to be 600 lbs. per square inch. The greatest live load will be produced by electric interurban cars weighing about 60 ton each. A uniform live load of 8600 lbs. per linear foot of span, or 4300 lbs. per linear foot for each rib, was assumed in making the calculations. This load of 8600 lbs. per foot is more than twice as great as Cooper's E40 loading which is used for most railroad bridges. When we consider that this enormous live load is less than 9 per cent of the dead load we get some idea of the size and weight of this arch. The stresses were determined for the following conditions of loading: (1) dead load, (2) dead load plus live load, and (3) dead load plus live load over the right

half of the span. Temperature stresses and stresses due to shortening from thrust were also computed. The arch was divided into 15 sections on each side of the center. The following notation is used.

y = distance from neutral axis of arch rib to span line of linear arch.

bv = ordinates in trial equilibrium polygon.

ak = distance from neutral axis to closing line of nuetral axis polygon.

ck = distance from true equilibrium polygon to closing line of neutral axis polygon.

Eccentricity = ck - ak.

m = distance from trial equilibrium polygon to span line of polygon.

n = distance from the trial closing line n n, which is parallel to span line of trial polygon, to the diagonal between these two line.

e = 58.02 ft.,the constant distance between the span line of linear arch and the closing line of the neutral-axis polygon.

x = distance from center of span to any section.

The trial pole distance H' was taken at 5,000,000 lbs. in each case.

For dead load plus half line load,

$$\overline{X} = \frac{\Sigma bv \times X}{\Sigma bv} = \frac{1083}{2090} = 0.52 \text{ ft.}$$

$$R' = \Sigma bv = 2090, \qquad T = \frac{R'}{2} = \quad 1045$$

$$n = \frac{R'}{31} = \quad 76.43 \text{ ft.}$$

$$\bar{x}_r = \bar{x}_l = \frac{\Sigma nx}{T} = \frac{35497}{1045} = 33.95 \text{ ft.}$$

$$T_l = \frac{\bar{x}_l - \bar{x}}{\bar{x}_l}\, T, \qquad T_r = \frac{\bar{x}_r + \bar{x}}{\bar{x}_r}\, T$$

$$m_l = \frac{T_l}{T}\, n = \frac{\bar{x}_l - \bar{x}}{\bar{x}_l}\, n = \frac{33.95 - 0.52}{33.95} \times 67.43 = 66.4 \text{ ft.}$$

$$m_r = \frac{T_r}{T}\, n = \frac{\bar{x}_r + \bar{x}}{\bar{x}_r}\, n = \frac{33.95 \quad 0.52}{33.95} \times 67.43 = 68.45 \text{ ft.}$$

$$\frac{\Sigma my}{\Sigma y^2 - e\Sigma y} = \frac{15848}{117601 - (58.02 \times 1799)} = \quad 1.196$$

True pole H = trial pole times $\dfrac{\Sigma my}{\Sigma y^2 - e\Sigma y}$

$$= 5{,}000{,}000 \times 1.196 = 5{,}980{,}000 \text{ lbs.}$$

For dead load plus full live load and for dead load alone, the loading was symmetrical about the center of the arch and the calculations for the true pole distance were much simpler. For dead load plus full live load,

$$m = \frac{\Sigma bv}{31} = \frac{10925 \times 2}{31} = 70.5 \text{ ft.}$$

$$\frac{\Sigma my}{\Sigma y^2 - e\Sigma y} = \frac{16574}{13240} = 1.244$$

True pole distance H = 5,000,000 x 1.244 = 6,220,000 lbs.

For dead load,

$$m = \frac{\Sigma bv}{31} = \frac{199620}{31} = 64.39 \text{ ft.}$$

$$\frac{\Sigma my}{\Sigma y^2 - e\Sigma y} = \frac{14966}{13240} = 1.13$$

Pole distance = 5,000,000 x 1.13 = 5,660,000 lbs. The temperature stresses were calculated from the formula,

$$Q = \frac{E\,l\,e\,t}{\Sigma y^2 - e\,\Sigma y} \times \frac{I}{S}$$

in which

E = 2,000,000 lbs. per sq. in., or 288,000,000 lbs. per sq. ft.

l = 280 ft.

e = 0.000006 = coefficient of expansion of concrete.

t = ±30 degrees F.

$\frac{I}{S} = 62.33$

From this Q = 68,300 lbs. and the stress at any section, omitting the stress due to the tangential component of Q, is given by the equation

$$f = \frac{6\,Qak}{Ad}$$

A = area of section.

d = depth of section.

The stresses due to the tangential component of Q are small and their use would be an unnecessary refinement, since the inaccuracies in the eccentric stresses due to loads are larger than this. The stresses due to a wind pressure are omitted. The stresses due to a wind pressure of 30 lbs. per sq. ft. of exposed surface are only about 20 lbs. per sq. in. of section of rib. In order to make allowance for slight inaccuracies and to be on the side of safety the stresses due to shortening of the arch were taken as 85 per cent of the temperature stresses, which is con-

siderably larger than the values ordinarily used. The direct stresses as well as all maximum and minimum stresses are compressive and there is no tension at any point in the arch. Plate XI is a reduced copy of the stress sheet. Tables 1 to 7 give the data for finding the true poles, the stresses under three conditions of loading, the temperature stresses, and the maximim and minimum stresses.

TABLE I

Data for making $S \div I$ Constant

Division	Dimensions of Ring, ft.		Horizon Division of Ring, ft.	S, ft.	$I = \frac{bd^3}{12}$	$S \div I$	Area of Section, sq. ft.
	Depth	Width					
P_1	6.00	18.06	5.2	5.2	325	.0160	108.4
P_2	6.05	18.20	5.3	5.3	336	.0158	110.0
P_3	6.10	18.32	5.5	5.5	352	.0158	111.8
P_4	6.15	18.46	5.7	5.8	358	.0160	113.5
P_5	6.25	18.60	5.9	6.0	378	.0158	116.2
P_6	6.35	18.75	6.4	6.5	400	.0162	119.0
P_7	6.50	18.92	6.8	7.0	434	.0161	123.0
P_8	6.70	19.09	7.2	7.5	478	.0158	127.8
P_9	7.00	19.29	8.2	8.8	550	.0158	135.0
P_{10}	7.35	19.53	9.4	10.1	646	.0156	143.5
P_{11}	7.75	19.81	11.0	12.0	770	.0156	153.5
P_{12}	8.20	20.41	12.8	14.9	925	.0161	165.0
P_{13}	8.70	20.54	14.6	18.0	1130	.0159	178.5
P_{14}	9.30	20.99	17.8	23.8	1405	.0169	195.0
P_{15}	9.90	21.68	18.2	29.4	1755	.0167	214.8

TABLE 2

Table of Dead and Live Loads in Kips

Division	D.L. per Rib	Summation of D.L.	L.L. per Horizontal Division	Summation D.L. + L.L.
P_1	153.8	153.8	22.4	172.6
P_2	160.1	313.9	22.8	359.1
P_3	168.1	482.0	23.8	550.9
P_4	177.7	659.7	24.5	753.1
P_5	191.1	850.8	25.4	669.6
P_6	211.4	1062.2	27.5	1208.5
P_7	237.0	1299.2	29.2	1474.7
P_8	272.5	1571.7	31.0	1778.2
P_8'	287.1	1858.8	61.8	2127.1
P_9	189.2	2048.0		2316.3
P_{10}	236.0	2284.0		2552.3
P_{11}	300.6	2584.0		2852.9
P_{11}'	511.0	3095.6	107.5	3471.4
P_{12}	388.2	3483.8		3859.6
P_{13}	513.5	3997.3		4373.1
P_{13}'	649.0	4646.3	107.5	5129.6
P_{14}	731.0	5377.3		5860.6
P_{14}'	893.0	6270.3	107.5	6861.1
P_{15}	961.0	7231.3		7822.1

TABLE 3

Data for finding True Equilibrium Polygon for Dead Load plus Live Load over Right Half of Span.

Section	y	y^2	ak	bv L	bv R	Diff. of bv Ordi.	x	x·bv	n L	n R
S_0	36.8	2684.5	+15.6	42.02	42.02	0.0	0.0	0.0	16.9	16.9
S_1	73.6	5417.0	+15.6	85.88	86.20	0.32	5.2	1.6	32.5	35.0
S_2	73.4	5387.5	+15.4	85.52	86.06	0.54	10.5	5.7	31.2	36.2
S_3	73.0	5329.0	+15.0	84.83	85.75	0.92	16.0	14.7	29.9	37.6
S_4	72.4	5241.7	+14.4	84.02	85.08	1.05	21.7	22.8	28.5	39.0
S_5	71.6	5126.6	+13.6	82.95	84.25	1.30	27.6	35.8	27.1	40.4
S_6	70.4	4952.1	+12.4	81.57	83.03	1.46	34.0	49.6	25.6	41.9
S_7	68.8	4733.4	+10.8	79.67	81.43	1.76	40.8	72.0	23.9	43.5
S_8	66.9	4475.6	+8.9	77.40	79.24	1.84	48.0	88.2	22.2	45.3
S_9	64.4	4147.4	+6.4	74.13	75.77	1.64	56.3	92.1	20.2	47.3
S_{10}	60.9	3708.8	+2.9	69.75	71.55	1.80	65.7	118.3	17.9	49.5
S_{11}	56.0	3136.0	−2.0	63.83	65.67	1.84	76.7	142.0	15.2	52.2
S_{12}	49.0	2401.0	−9.0	55.20	56.94	1.74	89.4	155.5	12.2	57.3
S_{13}	39.0	1521.0	−19.0	43.14	44.76	1.62	104.0	168.5	8.7	58.8
S_{14}	23.1	533.6	−34.9	24.92	25.88	0.96	121.8	117.0	4.3	63.1
S_{15}	0.0	0.0	−58.02	0.0	0.0	0.0	140.0	0.0	0.0	67.4

TABLE 3 Continued

Data for finding True Equilibrium Polygon for Dead Load plus Live Load over Right Half of Span

Sec.	Diff. of n Ordi.	nx	m		Σm	my	cK	
			L	R			L	R
S_0	0.0	0.0	+ 9.3	+ 9.4	+ 18.7	+1378	+15.6	+15.6
S_1	2.5	13	+18.6	+18.8	+37.4	+2750	+15.6	+15.7
S_2	5.0	52	+18.2	+18.6	+36.8	+2700	+15.2	+15.6
S_3	7.7	123	+17.6	+18.2	+35.8	+2615	+14.7	+15.2
S_4	10.5	228	+16.8	+17.5	+34.3	+2485	+14.1	+14.6
S_5	13.3	367	+15.8	+16.7	+32.5	+2325	+13.2	+14.0
S_6	16.3	554	+14.5	+15.4	+29.9	+2105	+12.1	+12.9
S_7	19.6	802	+12.7	+13.7	+26.4	+1820	+10.6	+11.5
S_8	23.1	1108	+10.5	+11.5	+22.0	+1470	+8.8	+9.6
S_9	27.1	1525	+7.1	+8.0	+15.1	+970	+5.9	+6.7
S_{10}	31.6	2075	+2.8	+3.7	+6.5	+395	+2.3	+3.1
S_{11}	37.0	2835	−2.9	−2.2	−5.1	−285	−2.4	−1.8
S_{12}	45.1	4025	−11.6	−11.2	−22.8	−1115	−9.7	−9.4
S_{13}	50.1	5200	−23.3	−23.5	−46.8	−1830	−19.5	−19.6
S_{14}	58.8	7175	−41.6	−42.5	−84.1	−1935	−34.8	−35.5
S_{15}	67.4	9415	−66.4	−68.5	−134.9	−00	−55.5	−57.2

TABLE 4

Stresses for Dead Load plus Live Load over Half of Span.

Sec.	Eccentricity		Stresses pounds per sq. in.					
			Eccentric		Direct		Direct + Eccentric	
	L, ft.	R, ft.	L.	R.	L	R	L	R
							385	385
S_0	0.0	0.0	0.0	0.0	385	385	385	385
							381	419
S_1	0.0	+ 0.1	0.0	± 38	381	381	381	343
							301	449
S_2	− 0.2	+ 0.2	∓ 74	± 74	375	375	449	301
							261	446
S_3	− 0.3	+ 0.2	∓ 110	± 75	371	371	481	296
							258	434
S_4	− 0.3	+ 0.2	∓ 106	± 70	364	364	470	294
							223	493
S_5	− 0.4	+ 0.4	∓ 135	± 135	358	358	493	223
							253	510
S_6	− 0.3	+ 0.5	∓ 97	± 160	350	350	447	190
							280	550
S_7	− 0.2	+ 0.7	∓ 60	± 210	340	340	400	130
							302	524
S_8	− 0.1	+ 0.7	∓ 28	± 194	330	330	358	136
							191	393
S_9	− 0.5	+ 0.3	∓ 125	± 75	316	318	441	245
							166	347
S_{10}	− 0.6	+ 0.2	∓ 134	± 45	300	302	434	257
							218	338
S_{11}	− 0.4	+ 0.2	∓ 77	± 38	295	300	372	262
							158	351
S_{12}	− 0.7	− 0.4	∓ 115	∓ 66	273	285	388	219
							207	380
S_{13}	− 0.5	− 0.6	∓ 75	∓ 90	282	290	357	200
							287	205
S_{14}	+ 0.1	− 0.6	± 13	∓ 76	247	281	261	357
							532	373
S_{15}	+ 2.5	+ 0.8	± 250	± 82	282	291	32	209

TABLE 5

Dead Load Stresses and Data for finding True Equilibrium Polygon under Dead Load.

Sec.	bv	m	my	ck	Eccentricity, ft.	Stresses, pounds per sq. in.		
						Eccen.	Direct	Eccen.+Dir't
S_0	41.0	+ 17.6	+ 649	+ 15.6	0	0	364	364 364
S_1	82.0	+ 17.6	+ 1295	+ 15.6	0	0	361	361 361
S_2	81.8	+ 17.4	+ 1276	+ 15.4	0	0	355	355 355
S_3	81.3	+ 16.9	+ 1233	+ 15.0	0	0	351	351 351
S_4	80.6	+ 16.2	+ 1173	+ 14.3	− 0.1	∓ 33	344	311 377
S_5	79.7	+ 15.3	+ 1096	+ 13.6	0	0	338	338 338
S_6	78.5	+ 14.1	+ 993	+ 12.5	+ 0.1	± 30	330	360 300
S_7	76.8	+ 12.4	+ 854	+ 11.0	+ 0.2	± 57	322	379 265
S_8	74.7	+ 10.4	+ 692	+ 9.2	+ 0.3	± 71	312	383 241
S_9	71.7	+ 7.3	+ 471	+ 6.5	+ 0.1	± 24	300	324 276
S_{10}	67.6	+ 3.2	+ 195	+ 2.8	− 0.1	∓ 21	285	264 306
S_{11}	62.0	− 2.4	− 134	− 2.1	− 0.1	∓ 18	282	264 300
S_{12}	53.7	− 10.7	− 522	− 9.4	− 0.4	∓ 61	268	207 329
S_{13}	42.2	− 22.2	− 865	− 19.6	− 0.6	∓ 85	272	187 367
S_{14}	24.4	− 40.0	− 924	− 35.4	− 0.5	∓ 60	265	205 325
S_{15}	0.0	− 64.4	− 00	− 57.0	+ 1.0	± 97	273	370 176

TABLE 6

Dead Load plus Full Live Load Stresses and Data for finding True Equilibrium Polygon.

Sec.	bv.	m	my	ck	Eccentricity ft.	Stresses, pounds per sq. in.		
						Eccen.	Direct	Direct + Eccen.
								480
S_0	45.1	+ 19.7	+ 726	+ 15.8	+ 0.2	± 80	400	320
								476
S_1	90.1	+ 19.6	+ 1443	+ 15.8	+ 0.2	± 79	397	318
								430
S_2	89.8	+ 19.3	+ 1417	+ 15.5	+ 0.1	± 39	391	352
								423
S_3	89.3	+ 18.8	+ 1372	+ 15.1	+ 0.1	± 38	385	347
								418
S_4	88.5	+ 18.0	+ 1303	+ 14.5	+ 0.1	± 38	380	342
								407
S_5	87.5	+ 17.0	+ 1217	+ 13.7	+ 0.1	± 34	373	339
								397
S_6	86.1	+ 15.6	+ 1098	+ 12.5	+ 0.1	± 33	364	331
								444
S_7	84.3	+ 13.8	+ 949	+ 11.1	+ 0.3	± 90	354	264
								426
S_8	81.9	+ 11.4	+ 763	+ 9.2	+ 0.3	± 83	343	260
								281
S_9	78.2	+ 7.7	+ 496	+ 6.2	− 0.2	∓ 50	331	381
								244
S_{10}	73.7	+ 3.2	+ 195	+ 2.6	− 0.3	∓ 70	314	384
								225
S_{11}	67.5	− 3.0	− 168	− 2.4	− 0.4	∓ 75	300	375
								165
S_{12}	58.4	− 12.1	− 593	− 9.7	− 0.7	∓ 122	287	399
								161
S_{13}	45.7	− 24.8	− 967	− 19.9	− 0.9	∓ 139	300	439
								343
S_{14}	26.4	− 44.1	− 1014	− 34.5	+ 0.4	± 53	290	237
								475
S_{15}	0.0	− 70.5	− 00	− 56.3	+ 1.7	± 180	295	115

TABLE 7

Temperature Stresses, Stresses due to Shortening of Rib, and Maximum and Minimum Combined Stresses

Section	Stresses, pounds per sq. in.			
	Temperature	Due to Shortening	Maximum	Minimum
			608	354
S_0	± 69	± 59	395	192
			602	351
S_1	± 68	± 58	391	192
			569	291
S_2	± 65	± 55	459	181
			561	252
S_3	± 62	± 53	490	181
			541	249
S_4	± 58	± 49	479	187
			589	215
S_5	± 52	± 44	501	127
			595	246
S_6	± 46	± 39	454	105
			618	274
S_7	± 37	± 31	406	62
			574	296
S_8	± 27	± 23	362	86
			426	188
S_9	± 18	± 15	444	210
			360	165
S_{10}	± 7	± 6	435	244
			339	211
S_{11}	∓ 4	∓ 3	382	261
			354	125
S_{12}	∓ 18	∓ 15	432	216
			386	102
S_{13}	∓ 32	∓ 27	398	195
			351	101
S_{14}	∓ 51	∓ 43	451	229
			542	248
S_{15}	∓ 66	∓ 56	331	22

Plate 1

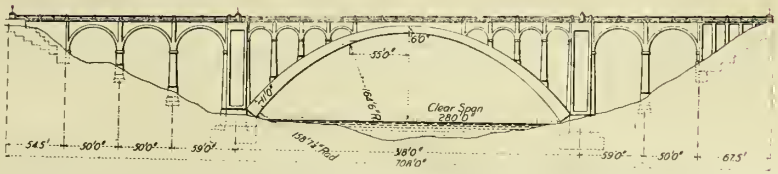
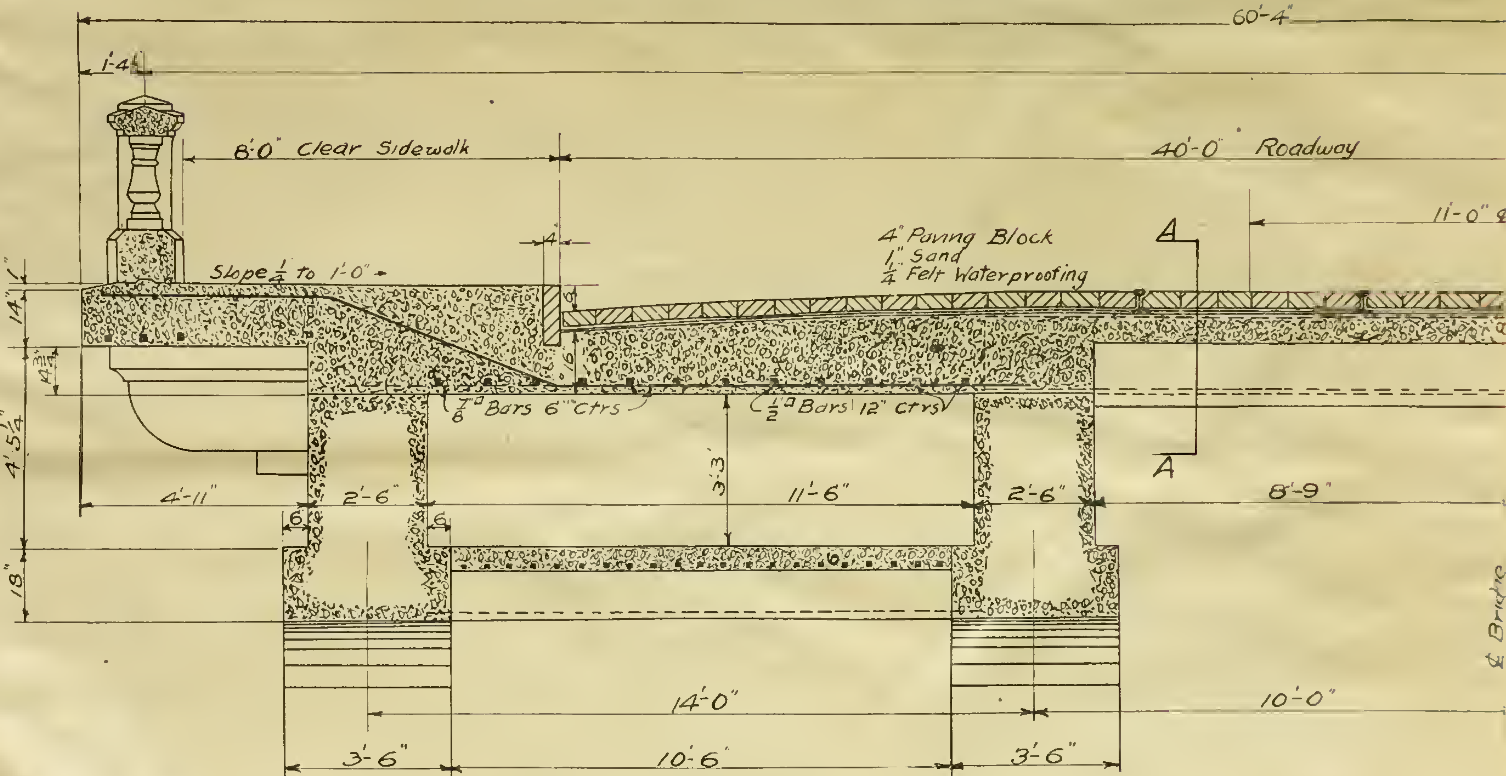


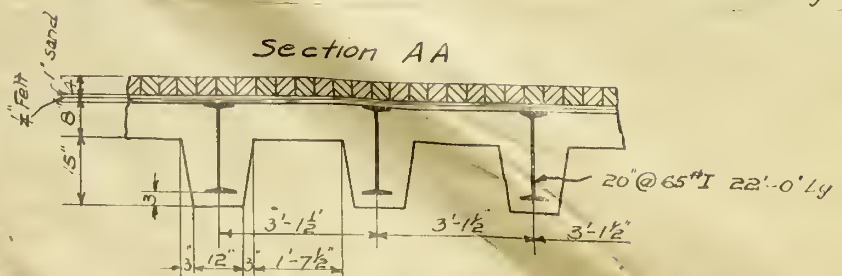
Fig. 1—Diagram Elevation of 280-Ft. Span Concrete Arch Bridge at Cleveland, O.

Plate II



Section Through Spandrel Arch

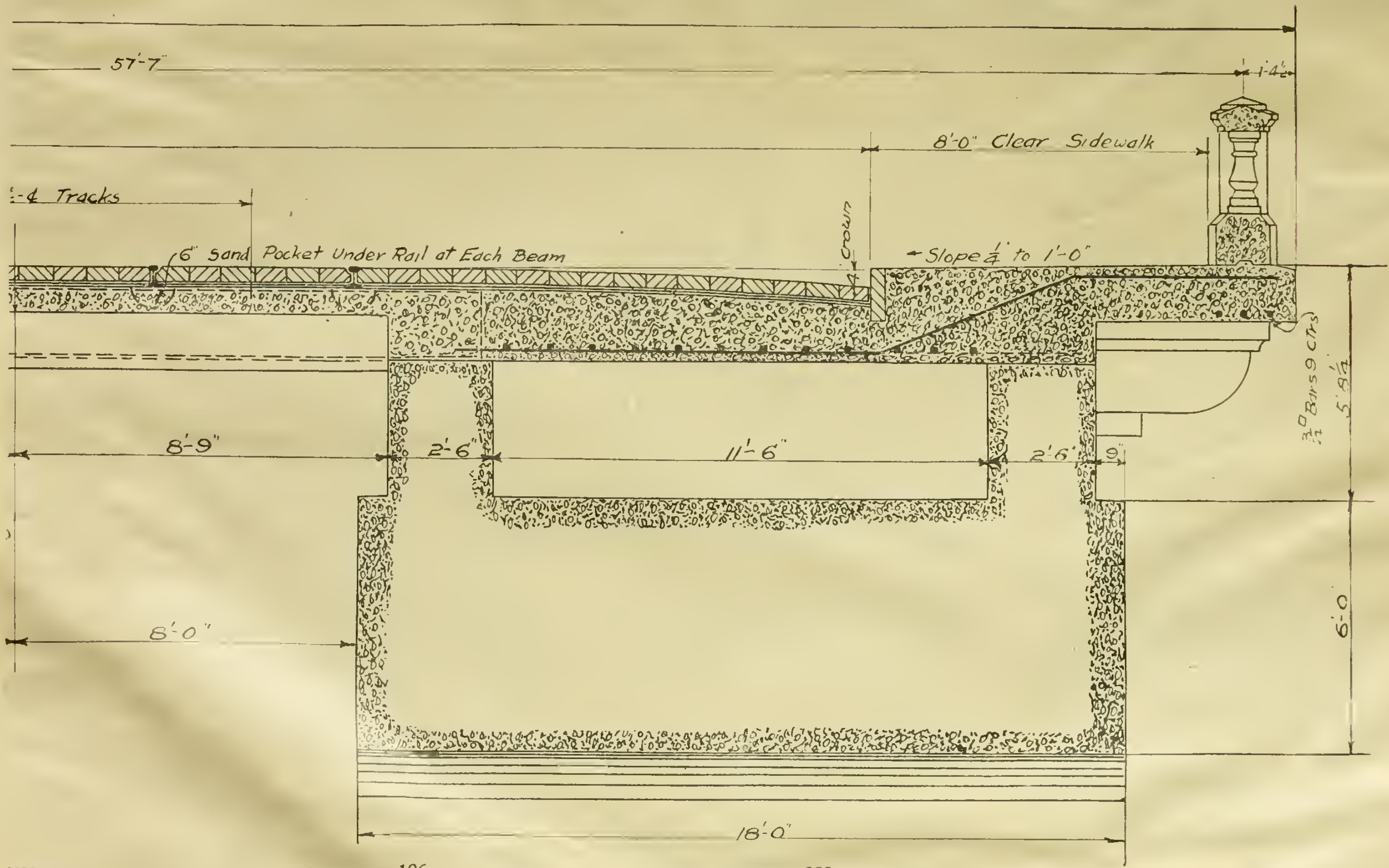
Section AA



DETAILS OF DESIGN. PROPOSED REINFORCED C

Fig. 2

Plate II



ONCRETE BRIDGE AT CLEVELAND. SEE PAGE 106. SECTION THROUGH 280-FT. ARCH.

Fig. 2

Plate III

Concrete Mixer and East Cableway Tower

Fig. 3

Plate IV

Fig. 4 Main Pier

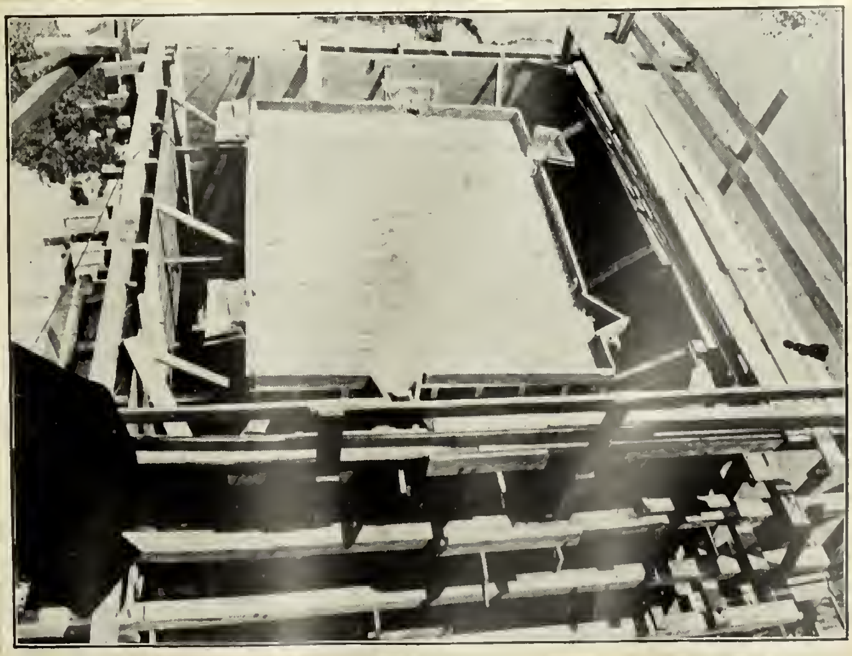
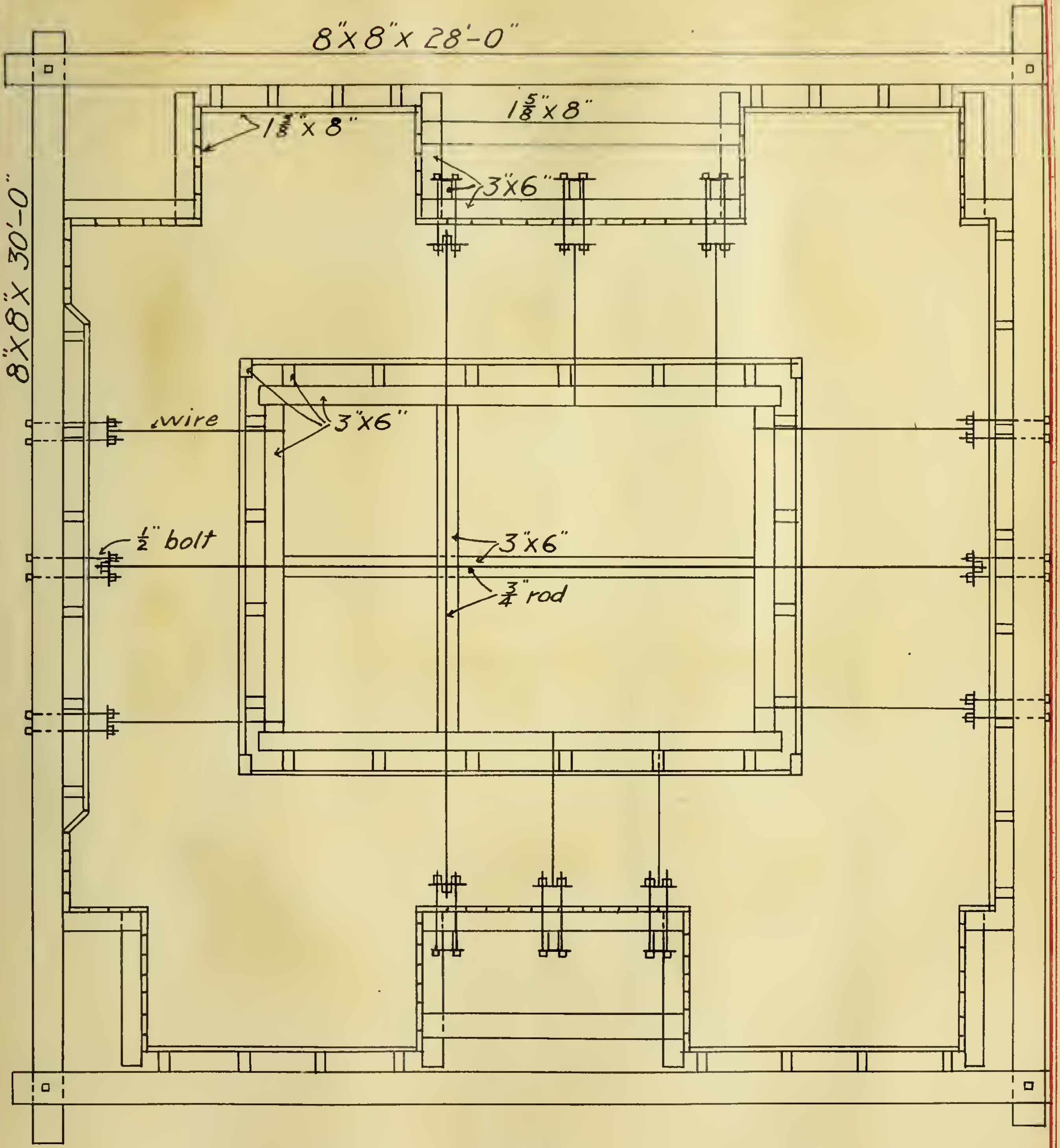


Fig. 5 Forms of Main Pier ready for the Concrete.

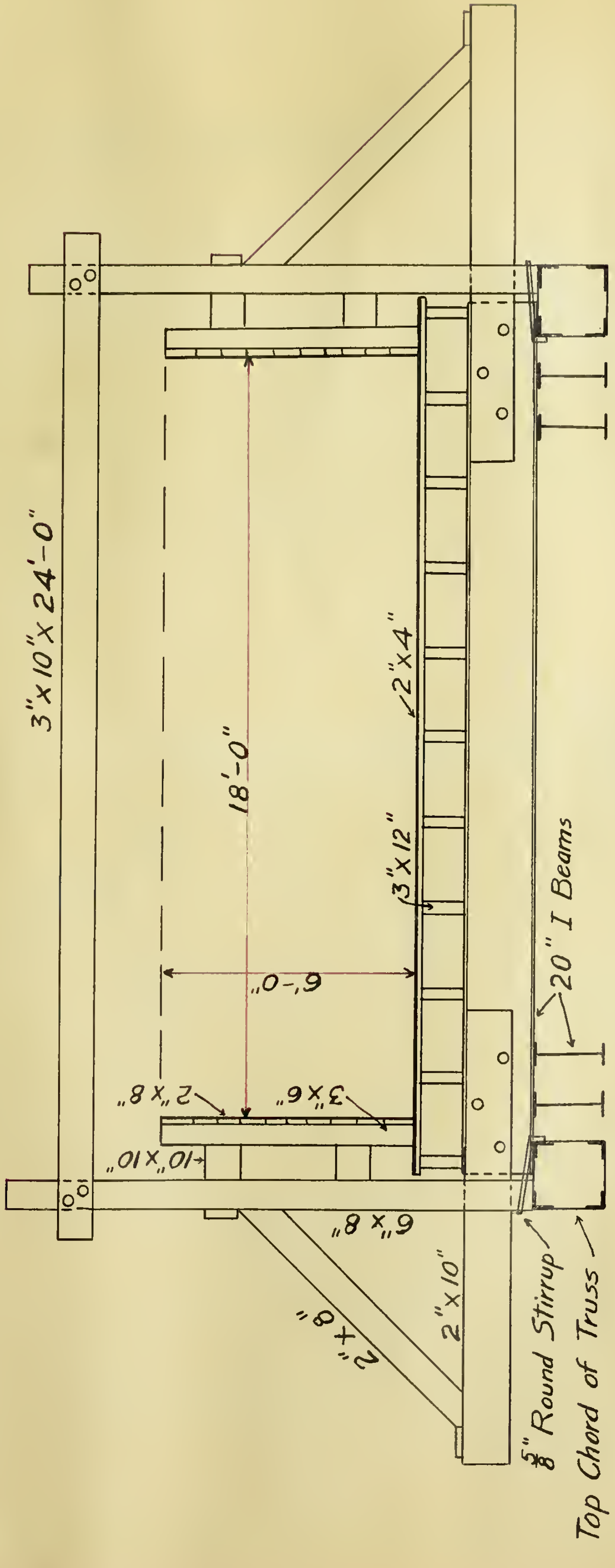
Plate V



Section of Forms Through Main Pier

Fig. 6

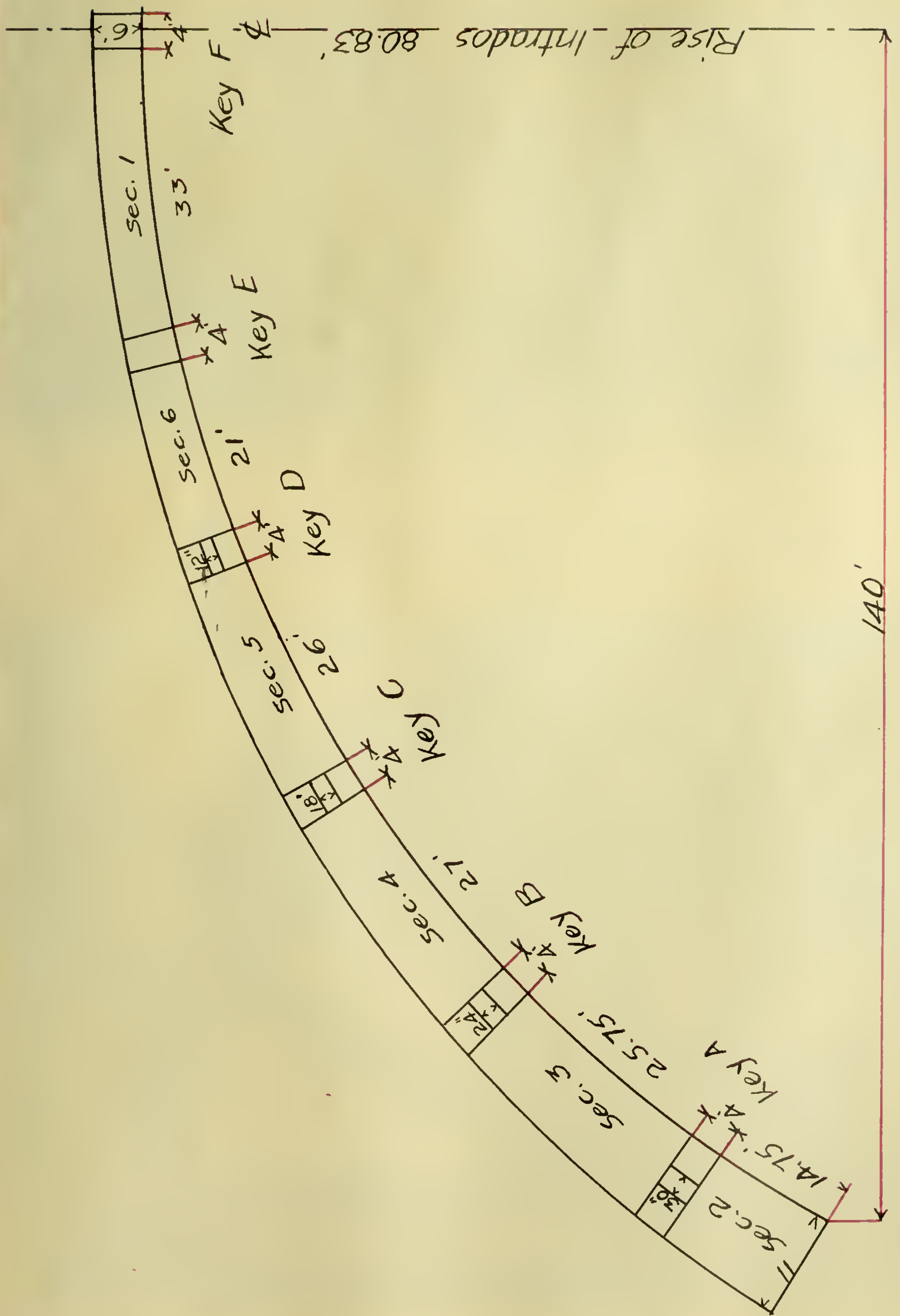
Plate VI



Section of Forms Through Crown of Main Arch

Fig. 7

Plate VII



Elevation of Half of Main Arch showing Keys and sequence of placing Concrete

Fig. 8

Plate VIII

Fig. 9 Steel Centers of Main Arch.

Fig. 10 Centres of Approach Spans

Plate IX

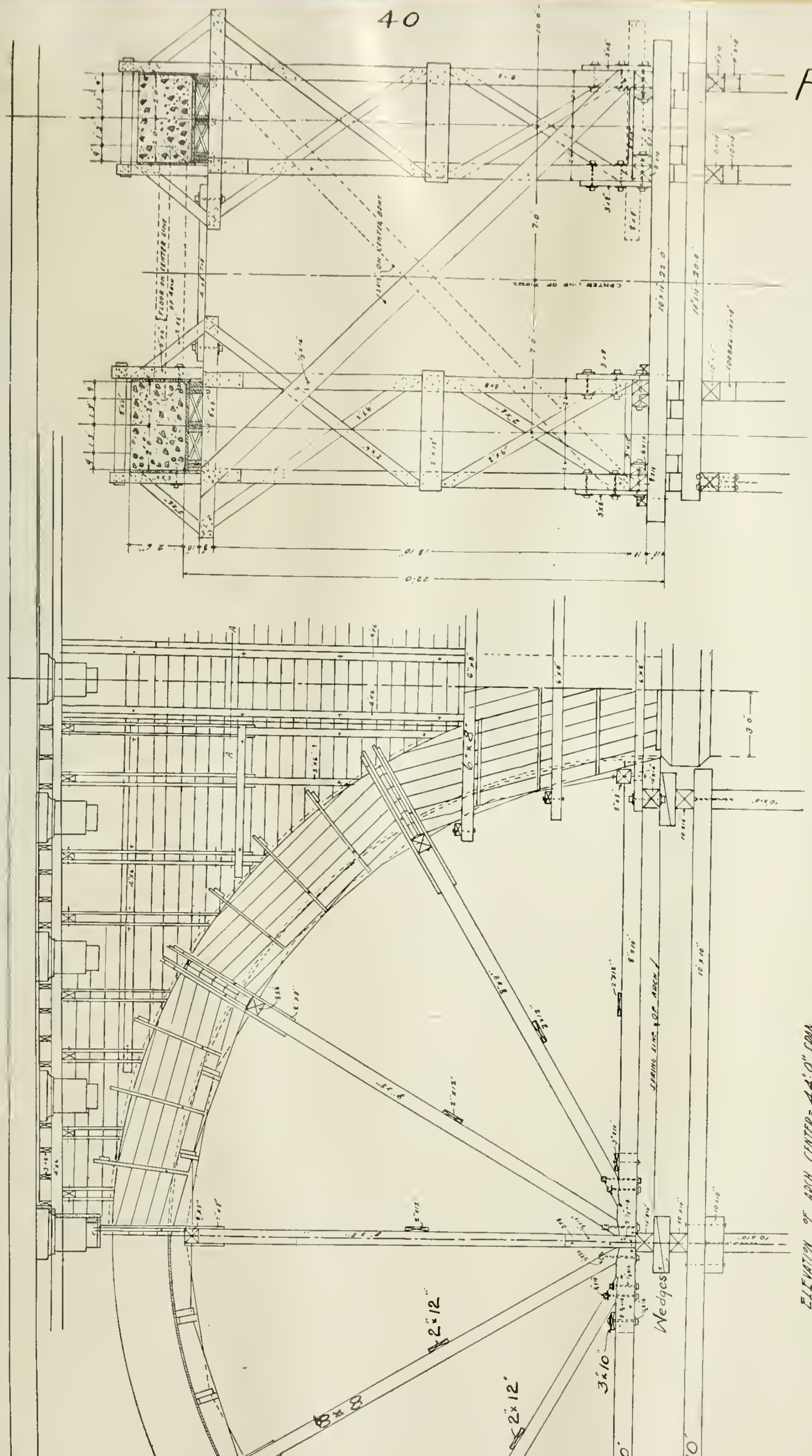


Fig. 11